4 Structural Performance

The BPAT inspected the structural performance of three primary construction types: reinforced concrete, reinforced masonry, and wood-frame. Inspections focused on the performance of single-family buildings. Isolated examples of success and failure in commercial buildings observed during field investigations were also documented.

It is important to state that wind speeds experienced on the island were not of the strength to test the reliability and adequacy of the reinforcing steel used in all of the reinforced and partially reinforced masonry walls. A more significant wind event striking Puerto Rico would likely have resulted in even more failures than were observed.

Planning Regulation 7 of Puerto Rico (building code) required strict practices for different primary construction types. Guidelines that were in place under Planning Regulation 7 for new construction accounted, at least partially, for wind and seismic loads, but these guidelines had not been consistently complied with or enforced effectively. Most of the damage the BPAT observed was directly related to design inadequacies and the lack of enforcement of Planning Regulation 7. Additional damage observed was related to poor quality of workmanship of self-built homes.

The 1987 amendment of Planning Regulation 7, which was in place at the time Hurricane Georges struck Puerto Rico, included wind speed design requirements to 110 mph (fastest-mile) for all buildings and design wind pressures for walls of 30 lbs. per square foot (psf) and for roofs up to 60 psf for residential buildings. Seismic provisions for commercial buildings and one- and two-family homes were also clearly identified. The failure to comply with and enforce this building regulation in all residential building construction resulted in widespread damages from Hurricane Georges. A major seismic event on the island could cause even more damage, since most of the elevated residential structures observed—even those that performed well during the hurricane—are not seismic resistant because they were constructed with inadequate lateral force resisting systems. The adoption and strong enforcement of the 1997 UBC should address many deficiencies observed by the BPAT.

In general, concrete/masonry structures performed well under the wind loading of Hurricane Georges. Structural damage to concrete and masonry structures from floodwater was usually limited to the building foundations as a result of erosion, scouring away of supporting soil, and the impact of waterborne debris.

Wood-frame structures generally performed poorly under wind loads generated by Hurricane Georges and damage was extensive throughout the island. A continuous load path from roof system to foundation was essential for building survival. Figure 4-1 illustrates a continuous load path for a wood-framed structure. The success of concrete and masonry structures illustrated the importance of a continuous load path while the failure in wood-frame structures illustrated the lack of proper wood construction techniques to provide an adequate and continuous load path. Figures 4-2 and 4-3 compare and contrast the success and failure of concrete and wood-frame building systems with similar wind exposure.

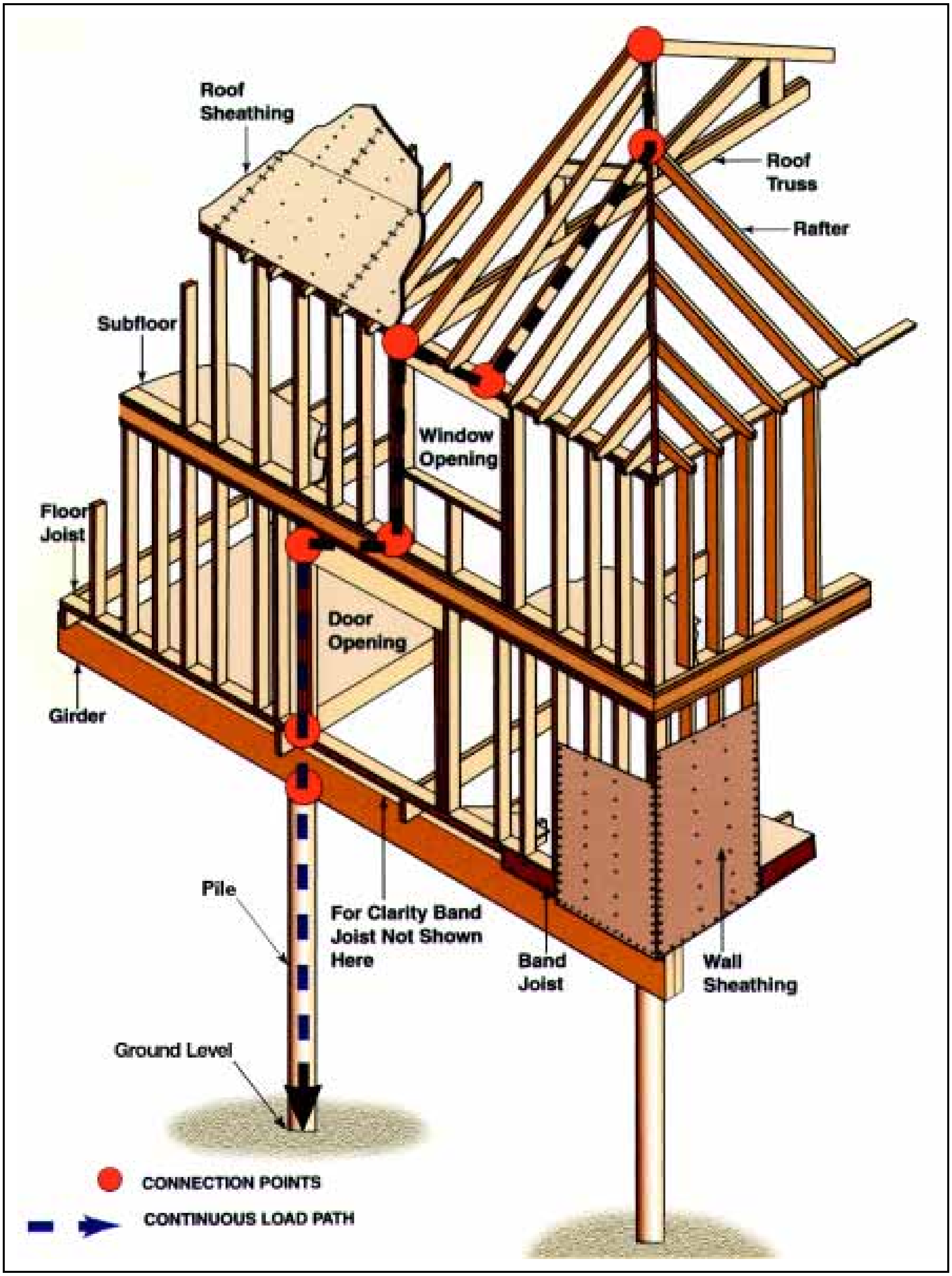

FIGURE 4-1 If a building has a continuous load path, forces and loads acting on any portion of the building will be transferred to the foundation of the building. This transfer occurs through building structural members (i.e., columns and beams) and the connections between these members. In this figure, the load path from the roof structure to the foundation is illustrated for an elevated, two-story wood-frame building.

FIGURE 4-2 A residential community constructed of concrete and masonry buildings with concrete roof structures. This community, located to the west of Luquillo experienced no complete building failures. The eye of the hurricane passed to the south of this community, placing it in the strongest wind quadrant of the hurricane.

FIGURE 4-3 A residential community constructed of wood-frame structures only. This community located to the north of Canóvanas, experienced significant structural damage and failure to almost all of its buildings. The eye of the hurricane also passed to the south of this community, which is located approximately the same distance from the path of the hurricane as the community in Figure 4-2.

Residential reinforced concrete/masonry structures with concrete roof decks performed well regardless of wind direction or velocity. Concrete/masonry structures with wood wall and roof framing generally performed poorly, regardless of siting. High velocity flood waters caused structural damage in SFHAs. Lower velocity floodwaters (also in SFHAs) inundated houses, causing considerable damage inside the buildings. Several concrete and masonry structures were left unstable from riverine and coastal erosion and mountain landslides (Figure 4-4).

FIGURE 4-4 Concrete residential structure with foundation damage caused by a landslide. Note unstable footings (circled).

4.1 Reinforced Concrete

The BPAT observed no structural damage to reinforced concrete residential or mid- and high-rise buildings. It was obvious that mid- and high-rise buildings received considerable attention from design professionals. Where concrete frames were observed, infill walls ranged from fully glazed to CMU (typically 6-in standard block) to metal and wood stud walls. Exterior cladding was stucco (trowel-applied cement plaster typically ½-in thick), Exterior Insulating Finishing Systems (EIFS), and block, brick, or stone veneer. These wall and cladding systems exhibited varying degrees of success or failure, as discussed in Section 5.2.

4.1.1 Reinforced Concrete Mid- and High-Rise Buildings

The lack of structural damage to reinforced concrete mid- and high-rise buildings was probably related to the role of the design professional in their construction as well as the fact that Hurricane Georges was not a design event. However, several buildings received considerable damage to the building envelope and are discussed in Section 5. The BPAT did not determine the seismic resistance of the mid- and high-rise buildings it observed.

4.1.2 Reinforced Concrete Essential Facilities

The BPAT inspected two fire stations, one in Adjuntas and the other on the island of Culebra, located approximately 20 miles east of the main island. Both fire stations had concrete roof decks. The stucco finish on both buildings prevented a direct observation of the wall systems that reportedly consisted of concrete columns with CMU infill. These structures also had open security grilles in the truck bays rather than large rolling doors. Neither station sustained structural damage during the hurricane. The Adjuntas fire station, which completed construction in 1998, featured a small percentage of exterior windows and an emergency electrical generator that was protected and enclosed within the building envelope (Figure 4-5). The BPAT was unable to determine the seismic resistance of either fire station.

FIGURE 4-5 Fire station in Adjuntas.

4.1.3 Concrete/Masonry Structures with Concrete Roof Decks

Reinforced concrete buildings (single-family homes) with reinforced concrete roof decks generally did not sustain structural damage (Figure 4-6). First floor walls in reinforced concrete residential buildings were usually 6-in to 8-in thick and constructed of reinforced concrete columns with masonry infill, or were solid concrete walls. CMU walls had varying amounts of reinforcement within the cells. Roof decks typically were flat and constructed of reinforced concrete. Many were exposed concrete with no roof covering. This structure type performed extremely well. Even buildings with unprotected wall openings did not experience structural damage.

The most significant damage observed for this type of construction centered around building envelope issues. Buildings (specifically single-family homes) typically had 4-in aluminum jalousie louvers (Miami windows) that were vulnerable to water infiltration during high wind events and allowed development of high internal pressure. Shutter systems are discussed in more detail in Section 5.4.

Residences constructed of reinforced concrete and a wood roof structure generally did not perform well during Hurricane Georges. Buildings without shutter systems were often breached, resulting in pressurization of the building and blown-off roofs. When shutters were observed to have been properly designed and installed, the roof framing and roofing typically were inadequate for lateral and uplift pressures, even without the added pressure from internal pressurization of the building.

FIGURE 4-6 Residential home constructed of reinforced concrete and masonry with a reinforced concrete roof deck in the mountains outside Adjuntas.

4.2 Masonry

The BPAT investigated a limited number of residential and nonresidential masonry buildings. Most of the buildings observed had wood-frame roof structures that were damaged during the hurricane (Figure 4-7).

FIGURE 4-7 Typical roof system failure between wooden roof system and concrete or masonry wall system.

4.2.1 Masonry Commercial Buildings

The BPAT observed several commercial buildings located on the island. Although many of them weathered the storm with minimal to no damage, this was mainly due to the siting of the buildings in areas of little wind and the buildings' relatively short un-reinforced masonry walls. The BPAT concluded that the commercial masonry buildings observed did not experience design level winds. Nonresidential buildings were observed with masonry wall systems and wood-framed roofs. Some roof failures in these buildings were the result of a poor connection between the wood-roof framing and the masonry walls. Termite damage was also observed in some residential wood-frame buildings, but the problem did not appear to be widespread. Figure 4-8 shows a termite-infested roof member that failed during the hurricane. The wood purlin and metal roof covering was separated from a building constructed with masonry walls and a wood-frame roof structure (Figure 4-9). Termite-weakened wood members were likely the starting point of this roof failure. Figure 4-10 is a close-up of the typical nailed connection between the purlins and the supporting rafters.

FIGURE 4-8 Termite-damaged wood purlin attached to metal roof panel. The entire roof system of this building failed and is shown in Figure 4-9.

FIGURE 4-9 Masonry wall church that lost roof purlins and its corrugated metal roof. Nails were the only connections used to resist wind loads. The gable ends of this church were unsupported except for purlins resting in the masonry.

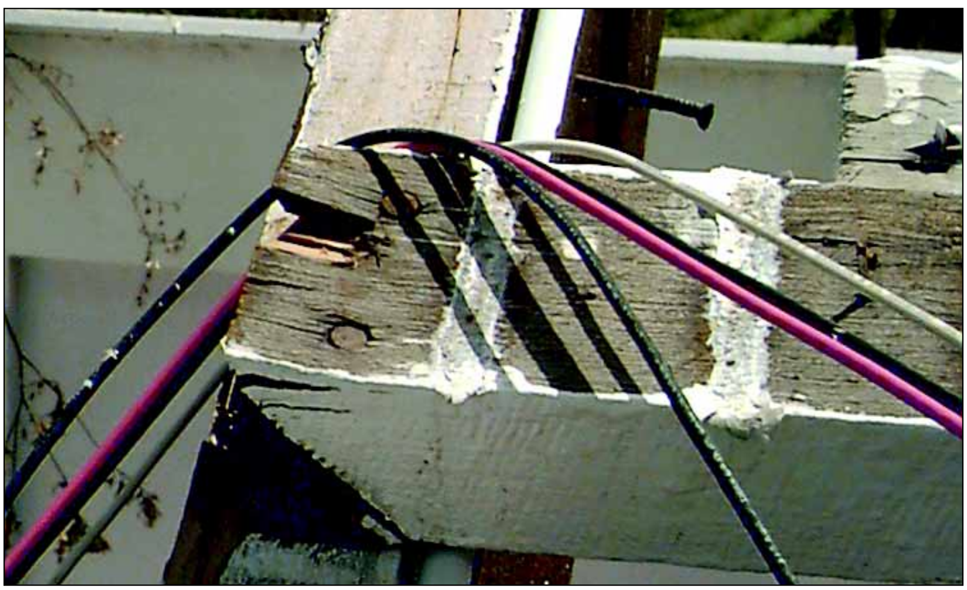

FIGURE 4-10 Nailed roof structure connection from church in Figure 4-9.

4.2.2 Residential Concrete/Masonry Structures with Wood-Frame Roof Structures

Successes and failures in masonry residential buildings were the same as those observed for concrete buildings. Success depended upon the existence of a continuous load path from the roof structure to the foundation for lateral and uplift loads. Conversely, wood-frame roof structures typically did not have a continuous load path to the foundation and widespread failure due to wind-induced uplift was observed. Figure 4-11 shows a typical nail withdrawal failure of a wood-frame roof/masonry wall connection.

FIGURE 4-11 Typical nail withdrawal failure in a wood-frame structure supported by a masonry wall with little uplift capacity at the connection.

Rafters ranged from nominally sized lumber, 2-in by 4-in or 2-in by 6-in that spanned 10 feet to 16 feet, and were spaced from 2-feet to 4-feet on center. Rafters were typically toe-nailed to the sill plate and not connected with hurricane clips or straps. The ridge rafters bore on a ridge beam (although sometimes the ridge beam was omitted). No connection other than nailing was generally made at the ridge line. Self-built trusses were also used. Similar to rafters, these trusses were connected only by nails to the sill plate. These trusses were sometimes manufactured by nailing the truss members together by toe-nailing, or by use of nominal 1-in lumber, or plywood for gusset plates. These self-built trusses were inadequate for the wind loads. As a result, widespread wood-frame roof failures were observed (Figure 4-7).

Corrugated metal was commonly used as a roof covering, typically fastened to nominal 1-in boards or 2-in by 4-in boards used as nailers to the rafters. Nailers were generally attached with two nails (16 penny or smaller) at the rafters. The trusses were generally unbraced or minimally braced for lateral loads and had little or no shear capacity from lateral loads. The attachment of the nailers for the corrugated metal roofing was completely inadequate for the uplift loads on the roofing. Since the majority of these homes had Miami windows, considerable internal pressures also acted on the roof system.

4.3 Wood-Frame Buildings

The BPAT investigated a number of residential wood-frame buildings. Very few of them survived the storm with little or no structural damage.

4.3.1 Commercial Wood-Frame Buildings

No new commercial buildings constructed from wood-framing were observed. As anticipated, many older, nonresidential buildings were damaged due to the lack of both uplift and lateral load paths from the roof system to the foundation.

4.3.2 Residential Wood-Frame Buildings

The BPAT observed many self-built single- and two-story residences. Self-built buildings are those that did not appear to be built to commonly accepted building practices. Very few appeared to have been designed or constructed to the current building regulations. As a result, a large number of wood-frame houses were structurally damaged during the hurricane. Houses built to current building regulations or newer codes weathered the storm successfully with minimal structural damage, but again it is worth noting that most of these houses were not exposed to design wind conditions.

Some residents installed hurricane clips as part of their mitigation efforts after hurricanes in 1995 and 1996. Figure 4-12 shows positive mitigation efforts implemented in a wood-frame house on the island of Culebra in 1996, as observed after Hurricane Georges. Utilization of clips and straps, however, was not typical in wood-frame buildings in Puerto Rico.

FIGURE 4-12 Hurricane clips installed in a wood-frame house on Culebra.

Improperly sized and spaced lumber was used throughout the self-built homes inspected. Some lumber appeared to be salvaged. Even in homes where clips were used, they were often installed with the incorrect number and size of nail. Wind completely destroyed a building constructed by a contractor only two months before Hurricane Georges occurred.

Traditional and inadequate nailing techniques were used on this structure while state-of-the-art clips, brackets, and fasteners were found lying beneath the building. Clips and hangers that were used did not employ the proper nails and failure resulted (Figure 4-13).

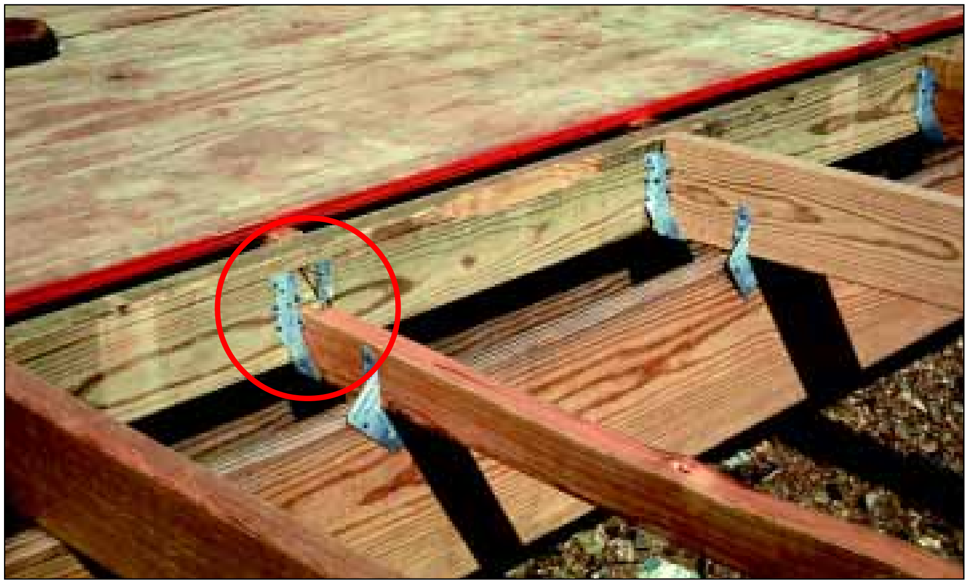

FIGURE 4-13 Example of the failure of wood member in the floor joist hanger due to the use of improper nails. The hurricane clip was used to secure the floor joist to the support beam. This house was located on the island of Culebra.

4.3.2.1 Residential Wood-Frame Walls

Framing layout and construction techniques used in almost all self-built wood-frame homes were not in compliance with Planning Regulation 7. Wall framing was constructed from nominal 2-in by 3-in and 2-in by 4-in studs. These studs were not properly supported laterally and not connected to the sill, bottom, or top plates with straps or connectors (Figure 4-14). The sill plate was inadequately (and often not) connected to the floor system with fasteners capable of resisting lateral and uplift forces. Top sill plates typically were single nominal 2-in by 4-in members that support the roof structure for gravity loading only. Nails appeared to be the primary connector used, with most connections being “toe-nailed.” End column (nominal 4-in by 4-in) members were observed in the wall sections of some wood-frame houses (Figure 4-15).

FIGURE 4-14 Example of wood-frame wall construction that failed during Hurricane Georges. This building was not constructed with any hurricane clips or straps.

FIGURE 4-15 Wood wall column that failed at connection to sill plate. This building was only two months old but was only partially constructed with clips, straps, and fasteners. The proper column fastener in the photograph was found unused beneath the house. This column was from the same house presented in Figure 3-13.

4.3.2.2 Residential Wood-Frame Roof Structures

The wood-frame roof structures discussed in Section 3.4 were found to be of poor quality construction and inadequately designed and constructed to withstand lateral and uplift wind forces. A majority of the wood-frame roof structures observed were gable ended; some of which had a peak with no ridge rafter.

Roofs were constructed of rafters, self-built trusses or pre-manufactured trusses. Rafter roof systems typically used nominal 2-in by 4-in to nominal 2-in by 6-in members. Lateral support or bracing was only provided by nominal 1-in by 3-in to nominal 1-in by 6-in nailers. Roof rafters and trusses were spaced on intervals ranging from 2-feet to 4-feet on center. Roof nailers for metal roof panels were observed on most wood-framing at 3-feet to 4-feet on center (Figure 4-16). Nailers did not typically provide adequate load capacity for the 110 mph design wind indicated in the 1987 amendment to Planning Regulation 7. In addition, the nailer/joist connections and the nailer/rafter connections observed generally were only connected with one or two nails. This simple nailed connection does not provide adequate resistance to shear and uplift forces that may be experienced during a high wind or seismic event. Figure 4-17 shows a typical self-built, wooden roof truss.

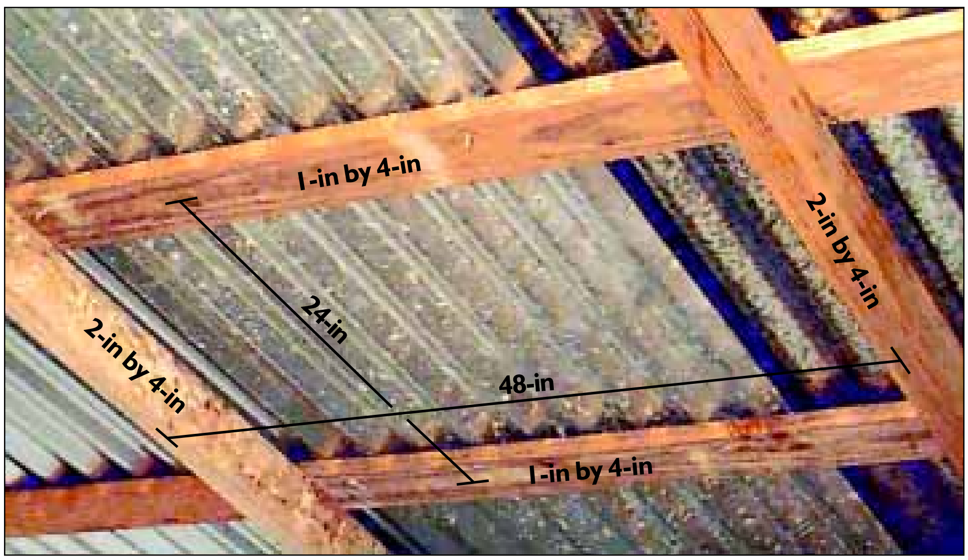

FIGURE 4-16 Typical wooden roof structure with metal roof panels above. Nails connected the metal panels to the nailers and the nailers were nailed to the joists.

FIGURE 4-17 Example of a self-built, wooden roof truss.

4.3.2.3 Residential Wood-Frame Floor Systems and Foundation Connections

The wood-frame buildings the BPAT inspected had varying floor systems and floor system-to-foundation connections. Many floor systems that remained in place after the wood-frame building constructed above was destroyed had minimal connections between the floor system and the foundation. Success of these connections is believed to be due to the failure of the roof and walls before the failure of these typically non-engineered connections between the floor system and the foundation (Figure 4-18). In a few homes, engineered floor system-to-foundation connections were observed (Figures 4-19 and 4-20).

FIGURE 4-18 Example of a non-engineered connection between the building foundation (concrete column) and the floor system. The wooden floor beam is connected to foundation rebar with an improper nailed connection.

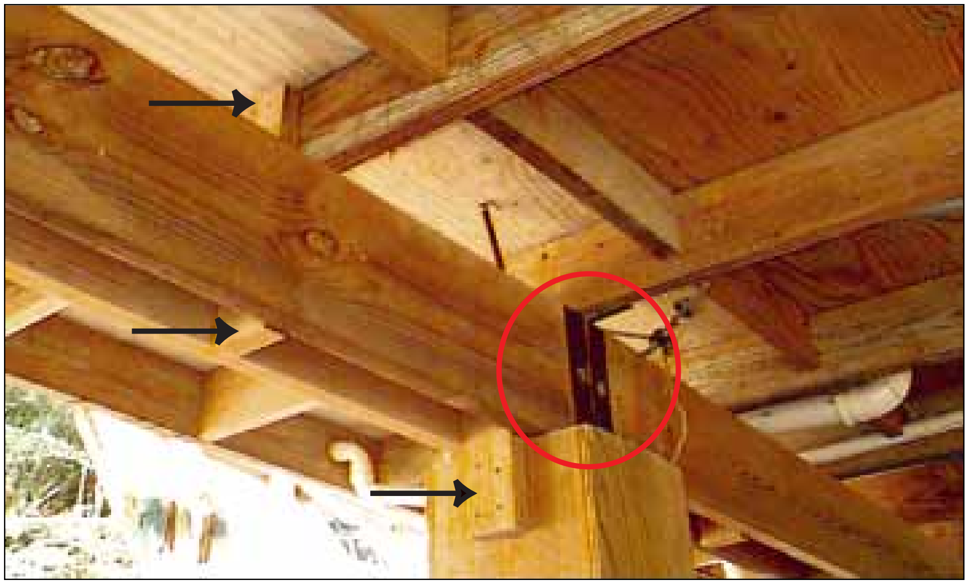

FIGURE 4-19 Example of an engineered connection between the floor beam and a concrete column (concrete column is enclosed in plywood). Vertical members (identified by arrows) provide continuous load path from floor joist to floor beams. Floor beams are connected to concrete columns with metal straps (circled). This house was located on the island of Culebra.

FIGURE 4-20 Example of a successful wood connection between support beam and floor joists. This is the same house shown above in Figure 4-19.

4.4 Hold-Down Cables

Tiedown or hold-down cables were used on some self-built wood-frame homes in Puerto Rico as a low-cost mitigation attempt. Typically, these cables were connected directly to the foundation of the structure although some cables were observed to have their own anchorage away and separate from the structure. Although there may have been exceptions, buildings with hold-down cables survived the effects of Hurricane Georges, but they remain largely untested during design wind conditions. In addition, there has been no engineering analysis of the effects of cable tiedown systems on load paths and structural and nonstructural building components. Hold-down cables are not expected to be effective unless the cables are designed and installed by an engineer or architect.

A majority of hold-down cables observed crossed over the ridge-line of the roof of the house at 10-foot spacing. A smaller percentage was observed running parallel to roof ridge-lines at 4-foot to 6-foot spacing as illustrated in Figure 4-21. The hold-down cables ranged from single strand steel wire to multi-strand steel cables.

FIGURE 4-21 Wood-frame house with metal roof covering with hold-down cables that run parallel (see arrows) and perpendicular to the roof ridge line. This house was set atop a ridge that experienced significant winds. The lack of damage can be attributed to the extra care taken in fastening down the corrugated metal roofing. The strapping would not have prevented buckling of the roofing or uplift at the eave.

4.5 Structural Seismic Considerations

Seismic load designs for commercial buildings and one- and two-family homes were addressed in Puerto Rico's 1987 amendment to Planning Regulation 7. For one- and two-family homes, seismic design is required for structural elements, but is not for the engineering of nonstructural building elements. For commercial buildings, the amendment addressed both topics, structural and nonstructural seismic design.

Nonresidential buildings were not investigated for compliance with the 1987 amendment to Planning Regulation 7 and the current structural seismic guidelines of the 1997 UBC. One- and two-family homes, however, were investigated for their ability to sustain a seismic event. Inspections revealed that most of these homes constructed of concrete, masonry, and wood appeared to lack the lateral stability necessary to survive a design seismic event. Many residential buildings were constructed on piles and columns with no visible lateral bracing. Connections between foundation systems and the building did not appear to have moment capacity required to withstand lateral forces induced by a design seismic event (Figure 4-22). shows an elevated residential building with no lateral support bracing its long columns. Figure 4-23 is a close-up of one of the footings for the tall columns shown in Figure 4-22. This type of small footing "setting" atop a rock outcropping was typical for houses built on hillsides.

FIGURE 4-22 Residential building supported atop tall, unbraced concrete columns. This type of unbraced support column was common in many areas.

FIGURE 4-23 Footing for tall columns in Figure 4-22. This footing is not adequately anchored to the supporting rock to resist lateral forces that may be induced during a design seismic event.